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Performance Assessment for Reinforced Concrete Buildings with Soft First Stories

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Synopsis

This study focuses on the seismic performances of soft-first-story buildings, which are demanded especially in urban areas. Six-story reinforced concrete buildings are focused on, and the seismic responses of the soft-first-story structures and typical frame structures are statistically assessed based on the results of dynamic response analyses. The mean annual frequency of the maximum interstoty drift ratio exceeding the specified value is computed, and the mean annual frequencies for the plural values are shown as a seismic hazard curve for each case. Eventually the probabilities of the maximum interstory drift ratios exceeding safety limit states are computed and compared. The soft-first-story buildings with the yield strength coefficient of more than 0.7 showed the same level of safety in comparison with typical frame structures, on the condition that the same deformation capacities are given to the main structural members.

Keywords: soft-first-story building, yield strength, deformation capacity, maximum interstory drift ratio, safety limit state, probability

1. Introduction

The residential buildings that have open spaces in the first stories are in great demand especially in urban areas. Architecturally it is very reasonable to allot such open spaces for parking lots and so on. As a result, the first stories become soft and weak relative to the other upper stories, since the first stories are composed of only the columns although the residential stories are divided by the rigid walls. Structurally those unbalances are unhealthy, and the soft-first-story buildings are well known for being susceptible to collapse through past big earthquakes.

This study focuses on the design that controls the seismic responses of the first stories by the strengths and the deformation capacities. To assess the seismic performances of the structures, a probabilistic procedure is adopted based on the works by Shome and Cornell (1998) and Vamvatsikos and Cornell (2002). 'The probability that the seismic response exceeds the safety limit state is *x* % in 50 years', this is very clear and accessible to the residents. By comparing the performances of the soft-first-story buildings to those of the typical frame structures, the relative assessments for the soft-first-story buildings are also completed.

2. Procedure of assessment

2.1 Models of analyses

The six-story RC buildings shown in Fig. 1 are considered. The analyzed cases consist of three soft-first-story buildings and two frame structures. The properties and the dimensions of the buildings are shown in Table 1. One interior frame of each building is assumed to represent the performance of the building, and the portion corresponding to the one frame is extracted for modeling.

For the soft-first-story buildings, the yield strength coefficients (the base shear coefficients at which structures yield) of 0.35, 0.70 and 1.05 are selected. The

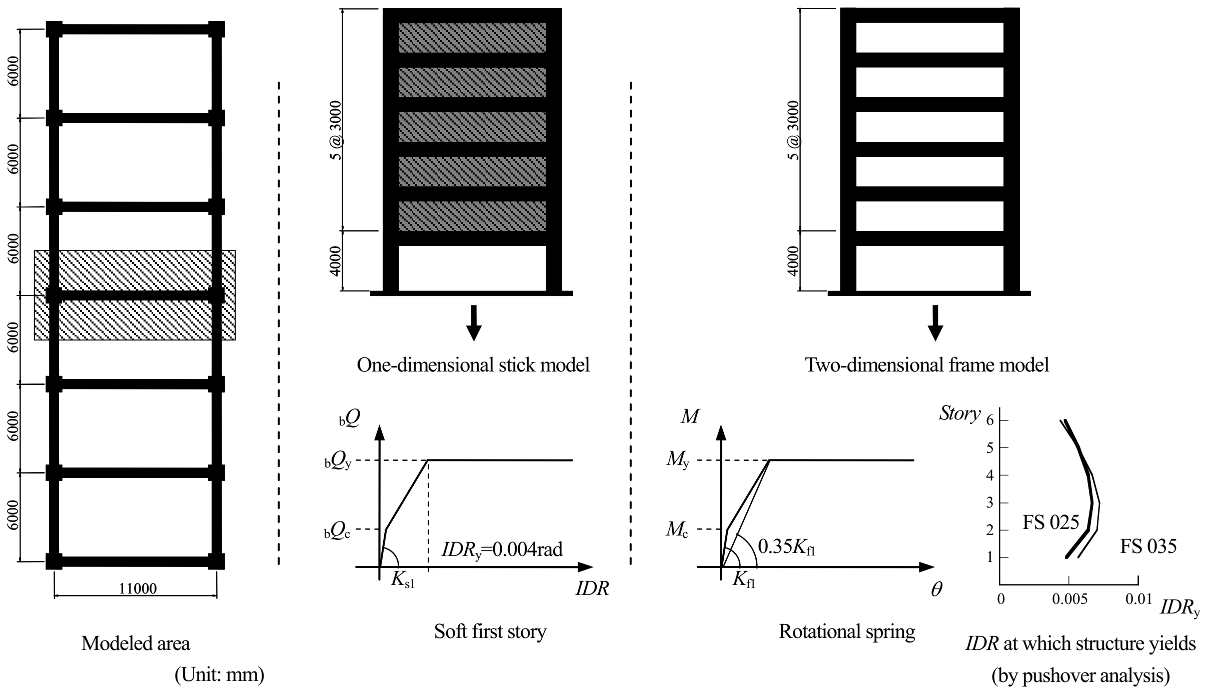


Fig. 1 Soft-first-story building and frame structure

structures are realized by stick models. The shear deformation and the flexural deformation of the each story are expressed by a shear spring and a rotational spring. The inelastic behaviors of the first stories are defined by applying Takeda model (Takeda et al. 1970) to the relationships between the shear force and the interstory drift ratio. The initial stiffness K_{s1} is calculated by Eq. (1) assuming the flexural deformation of the two columns.

$$K_{s1} = 2 \cdot \left(\frac{12EI}{l^3} \right) \cdot L \tag{1}$$

where E is young modulus, I is the moment of inertia of gross section, l is the clear span, L is the story height.

The crack strength ${}_bQ_c$ is calculated based on the flexural crack strength of the column, which is subjected to the tension by the overturning moment of the structure. The interstory drift ratio corresponding to the yield strength is defined as 0.004 rad regardless of the size of the column, based on the work by Arzpeima and Kuramoto (2003). The stiffness after the yield is defined as K_{p1} times 0.001.

For the frame structure, the yield strength coefficients of 0.25 and 0.35 are selected. The structures are realized by two-dimensional frame models. The beams and the bottom ends of the columns are modeled as inelastic elements, and by making the other columns infinitely strong, the complete mechanisms for the frame structures are guaranteed. The flexural deformations of the beams

Table 1 Assumed cases and dimensions of structural members

(1) Soft-first-story building

Cases	Yield strength coefficient	Columns of 1st stories	Columns of 2nd to 6th stories	Beams of 2nd - roof stories	Thickness of the walls	1st mode periods (sec.)
SB 035	0.35	900 x 900	900 x 900	600 x 1000	200	0.20
SB 070	0.70	1050 x 1050	900 x 900	600 x 1000	200	0.17
SB 105	1.05	1200 x 1200	900 x 900	600 x 1000	200	0.15

(2) Frame structure

Cases	Yield strength Coefficient	Columns of 1st to 6th stories	Beams of 2nd story	Beams of 3rd - 4th stories	Beams of 5th - roof stories	1st mode periods (sec.)
FS 025	0.25	900 x 900	600 x 1100	600 x 1000	600 x 900	0.70
FS 035	0.35	900 x 900	650 x 1200	650 x 1100	650 x 1000	0.62

(Unit of dimensions: mm)

are concentrated to the rotational springs at the ends. The initial stiffness K_{f1} of the rotational spring is calculated by Eq. (2).

$$K_{f1} = \frac{6EI}{l} \quad (2)$$

The inelastic behaviours of the rotational springs are defined by Takeda model. The secant stiffness corresponding to the yield is defined as K_{f1} times 0.35. The yield moments of the hinges are decided so that all the hinges simultaneously yield at the assumed base shear under the pushover analysis based on the linear load distribution. The stiffness after the yield is defined as K_{f1} times 0.001.

From the results of eigenvalue analyses, the natural periods of the soft-first-story structures are 0.15-0.17 and those of the frame structures are 0.62 and 0.70. The structures are damped by 5 % coefficient for the first mode, but the damping force of each member is changed in proportion to the instantaneous stiffness of the member. The *P-Δ* effects are considered using a geometric stiffness formulation.

2.2 Site Hazard and Ground Motion

The selected scalar to represent the intensity of ground motion (intensity measure IM) is the peak ground acceleration *PGA*, whose hazard curve is available in Recommendations for Loads on Buildings (Architectural Institute of Japan, AIJ, 1993). The seismic hazard curve is the relationship between *PGA* and the mean annual frequency of *PGA* exceeding the specified value. The reciprocal number of the mean annual frequency is the return period, *T* years, and the seismic hazard curve based on *PGA* is often referred by engineers and structural designers to express the return period of the ground motions considered.

In accordance with Eq. (3), the hazard curve $\lambda_{PGA(x)}$ is expressed by Eq. (4).

$$PGA = x = A_0 \left(\frac{1}{100 \lambda_{PGA}(x)} \right)^{0.54} \quad (3)$$

$$\lambda_{PGA}(x) = 0.000527 \cdot x^{-1.85} \quad (4)$$

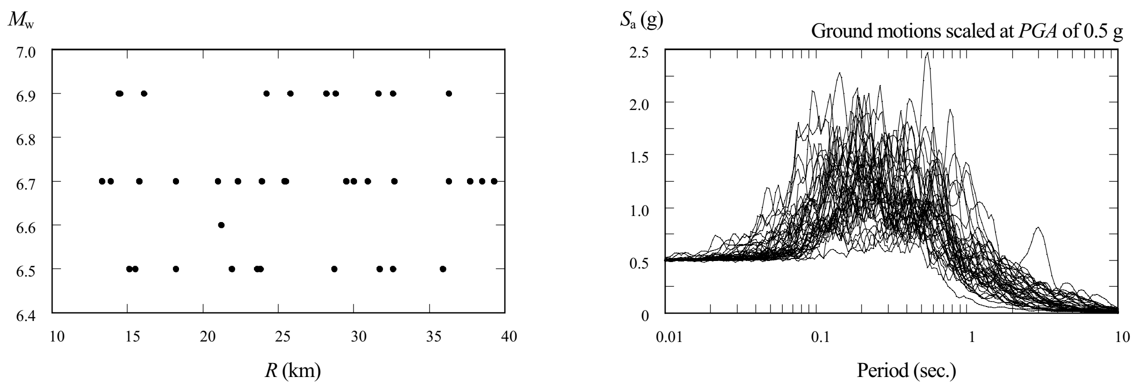
where A_0 is 200/980 (g).

A set of 40 ground motions (Medina 2003) is used for the dynamic response analyses. The ground motions were recorded in various earthquakes in California. The sites are categorized as Type-D site of NEHRP (183m/sec<V_s<366m/sec or 15<N<50, where V_s is shear wave velocity, N is N value of SPT test). The earthquake magnitudes are from 6.5 to 6.9, and the source-to-site distance ranges from 13 to 40 km, as shown in Fig. 2 (1). The earthquakes with the large magnitude and the small source-to-site distance are adopted to consider the safety limit states of the structures.

The acceleration response spectra of the selected 40 ground motions are shown in Fig. 2 (2). The ground motions are scales at *PGA* of 0.5 g. Thus, the frequency content of the ground motion cannot be considered explicitly. The large dispersion in spectral accelerations due to the different frequency content of the selected ground motions is illustrated in Fig. 2 (2).

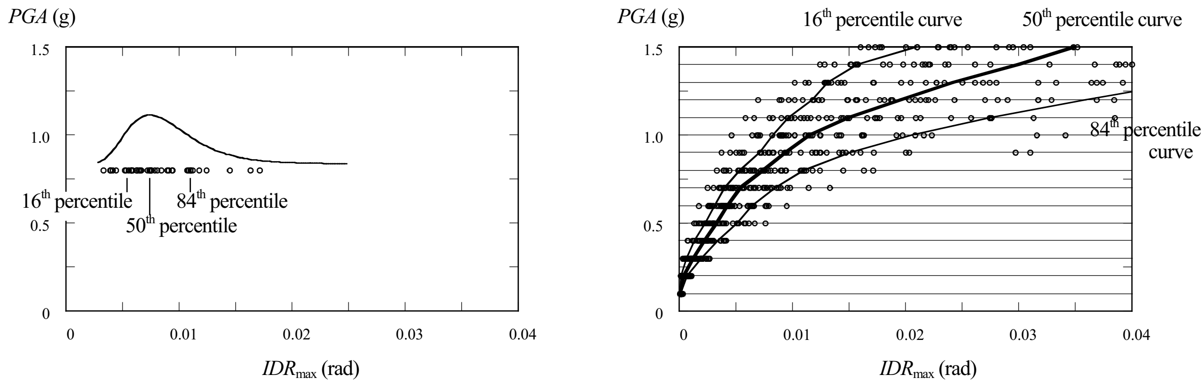
2.3 Statistics of Dynamic Responses

The 40 results of the dynamic response analyses at given a *PGA* are shown by the plots in Fig. 3 (1). The maximum interstory drift ratio IDR_{max} is adopted as an



(1) Moment magnitude and source-to-site distance (2) Acceleration response spectra (h= 5 %)

Fig. 2 Ground motions used for dynamic response analyses



(1) Responses at given a *PGA* (2) Continuous estimation of seismic response

Fig. 3 Statistics of seismic responses (SB 105)

engineering demand parameter EDP to represent the degree of seismic response. For the dispersed data, the statistics are conducted to the direction of EDP (to the lateral axis). Since the impacts of a few ground motions that produce extremely large deformations are apt to dominate the other results in the inelastic dynamic response analyses, the "counted" statistics are adopted. For the set of 40 ground motions, the average of the 20th and 21st sorted values is taken as median (50th percentile).

The median of the natural logarithm of the data $\ln(IDR_{max})^{50\%}$, and the equivalent dispersion δ_{eq} of the data are used as parameters in applying the lognormal distribution to the data. The equivalent dispersion δ_{eq} corresponds to the difference between the $\ln(IDR_{max})^{50\%}$ and the 16th percentile $\ln(IDR_{max})^{16\%}$, or the difference between the $\ln(IDR_{max})^{50\%}$ and the 84th percentile $\ln(IDR_{max})^{84\%}$. This is based on the assumption that the mean ± one sigma (the standard deviation) is the 16 percentile or the 84 percentile in the normal distribution. Thus, δ_{eq} is calculated by Eq. (5).

$$\delta_{eq} = \frac{\ln(IDR_{max})^{84\%} - \ln(IDR_{max})^{16\%}}{2} \tag{5}$$

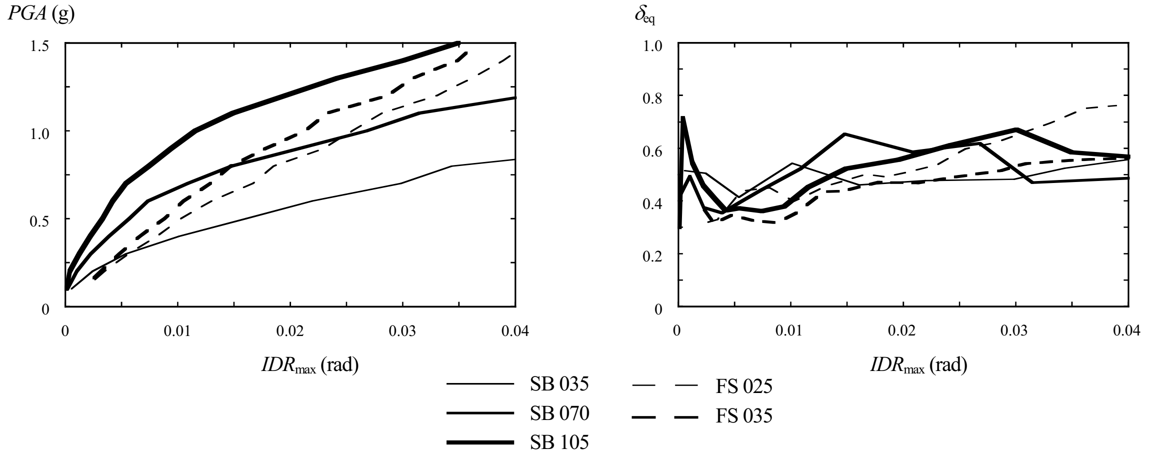
It should be noted that δ_{eq} is approximately the coefficient of variation of the data (Benjamin and Cornell 1970).

Fig. 3 (2) shows the results of the incremental dynamic analyses, in which *PGA* is increased by 0.1 g. Based on the statistical estimation for each level of *PGA*, the distribution of the responses are identified continuously. The 16th percentile, 50th percentile and 84th percentile curves are indicated with the bold lines.

3. Comparison of Performances

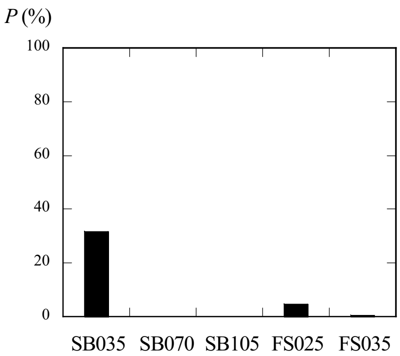
3.1 Seismic Response

Fig. 4 (1) shows the 50th percentile curves of the

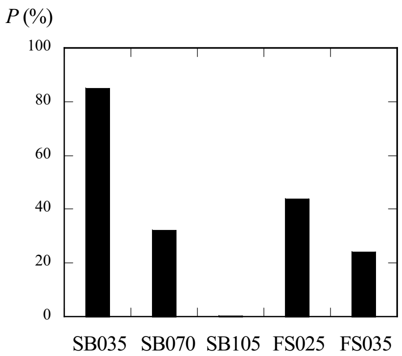


(1) 50 Percentile curves (2) Equivalent dispersions

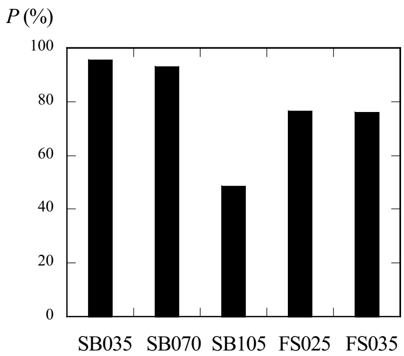
Fig. 4 Statistical evaluations of seismic responses



(1) Given *PGA* of 0.50 g
(Return period: 530 years)



(2) Given *PGA* of 0.80 g
(Return period: 1260 years)



(3) Given *PGA* of 1.20 g
(Return period: 2660 years)

Fig. 5 Conditional probabilities of exceeding 0.02 rad at Given *PGA*

responses (the interstorydrift ratio, $IDR_{\max}$), which represent the degree of vulnerabilities for *PGA*. In the case of the soft-first-story buildings, the seismic responses given *PGA* are effectively reduced by the increase of the yield strength. However, the slopes become small in the range of $IDR_{\max}$ over 0.04 rad, which corresponds to the yield, although the frame structures keep almost linear slopes. Thus, it is suggested that the slopes of the curves are influenced by the redundancies depending on the mechanism types. Fig. 4 (2) shows the relationships between δ_{eq} and 50th percentile of $IDR_{\max}$. Although the tendency that δ_{eq} slightly increase with the increase of the $IDR_{\max}$ can be observed, δ_{eq} are between 0.3 and 0.7.

By using the two parameters mentioned above, the conditional probability that $IDR_{\max}$ exceeds *idr* at given *PGA* is obtained by Eq (6).

$$P[IDR_{\max} > idr] = 1 - P[IDR_{\max} \le idr]$$

$$= 1 - \Phi\left(\frac{\mathrm{Ln}(idr) - \mathrm{Ln}(IDR_{\max})^{50\%}}{\delta_{eq}}\right) \qquad (6)$$

Fig. 5 shows the conditional probabilities that $IDR_{\max}$ exceeds 0.02 rad at given *PGA*. The selected *PGAs* are 0.50, 0.80 and 1.20 g and corresponding to the return periods of 530, 1260 and 2660 years, respectively. It should be noted that the level of *PGA* considered in the code of Japan approximately corresponds to 0.50 g. In that level, the conditional probabilities are kept very small (less than 0.5 %) except for the soft-first-story building with the yield strength coefficient of 0.35 and the frame structure with the yield strength coefficient of 0.25. However, the conditional probabilities become high as *PGA* becomes large. In the case of *PGA* of 1.20 g, the probabilities that $IDR_{\max}$ exceeds 0.02 rad ascend to more than 49%. Thus, it is suggested that if the design calls for the lower value of the probability that $IDR_{\max}$ exceeds 0.02 rad given *PGA* of 1.20 g, the yield strength ratio of more than 1.05 is demanded in the case of the soft-first-story building.

The mean annual frequency that $IDR_{\max}$ exceeds the specified value *idr* is calculated by Eq. (7).

$$\lambda_{IDR\max}(idr) = \int_0^\infty P[IDR_{\max} > idr \mid PGA = x] \left| d\lambda_{PGA}(x) \right| \qquad (7)$$

Eq. (7) consists of the slope of the seismic hazard curve in Eq. (4), which means the mean annual frequency of

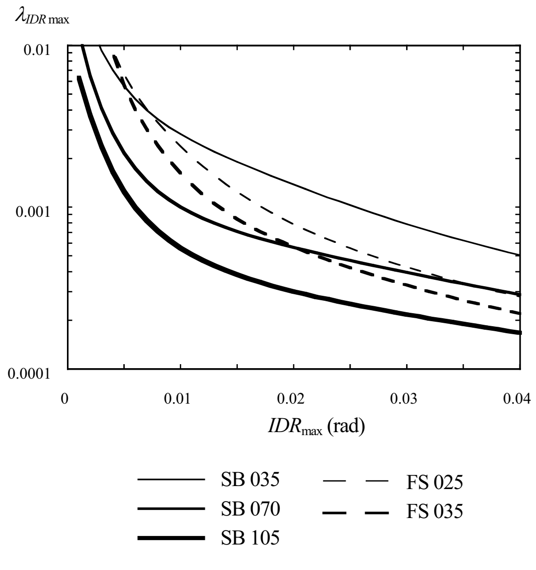


Fig. 6 Hazard curves of $IDR_{\max}$

the occurrence of the event with PGA of x, and the conditional probability of the response in Eq. (6). Fig. 6 shows the hazard curve for $IDR_{\max}$ derived from Eq. (7). The mean annual frequencies given $IDR_{\max}$ become smaller with the increase of the yield strength coefficient. For each, the mean annual frequency decrease as $IDR_{\max}$ becomes larger, but the slope of the descent is larger in the frame structures than in the soft-first-story buildings. This can be explained by the tendencies of 50th percentile curves in Fig. 4 (1).

3.2 Safety limit state

The method to estimate the ultimate deformation capacity and the verification are shown in the Design Guideline Based on Ultimate Strength Concept (AIJ, 1990). Fig. 7 shows the relationships between the test results and the estimations, which are verified in the guideline. The fragility curve of the ultimate deformation capacity $F(r)$ at given the estimated value is defined by applying the lognormal distribution to the test results at given the estimation. From the statistics for the ratios of the experimental results to the calculations in Fig. 7, the 50 percentile is obtained as 1.41, and the equivalent dispersion δ_{eq} is obtained as 0.39. For the soft-first-story building, the 50 percentile values are multiplied by l/L (where l is the clear span of the column, 3.0 m, L is the story height, 4.0 m) to convert the maximum rotational angle of the column to the maximum interstory drift ratio $IDR_{\max}$. For the frame structures, the maximum rotational angles of the beams are assumed to correspond to the maximum interstory drift ratios $IDR_{\max}$.

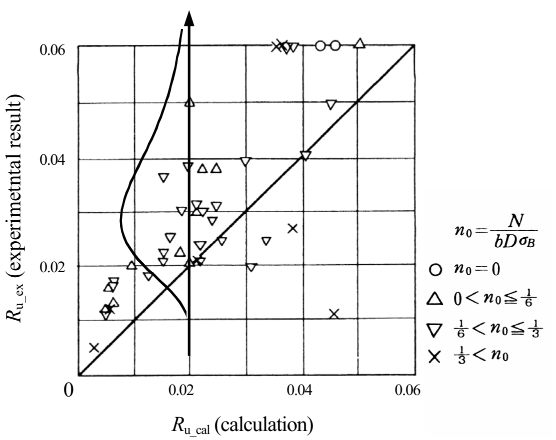


Fig. 7 Verification of estimated ultimate deformation capacities (AIJ, 1990)

Thus, the mean annual frequency that the response exceeds the safety limit state can be obtained by Eq. (8) that consists of the fragility curve based on interstory drift ratio $F(idr)$ and the slope of the hazard curve of $IDR_{\max}$ in Eq. (7).

$$\lambda_{Ru} = \int_0^\infty F(idr)\left|d\lambda_{IDR\max}(idr)\right| \qquad (8)$$

Fig. 8 shows the mean annual frequencies that the seismic responses exceed the safety limit state at given the estimation of the ultimate deformation capacity.

In addition, the mean annual frequencies can be converted to the probabilities, assuming Poisson process. That is, the probabilities of exceeding the safety limit state in t years are given by Eq. (9).

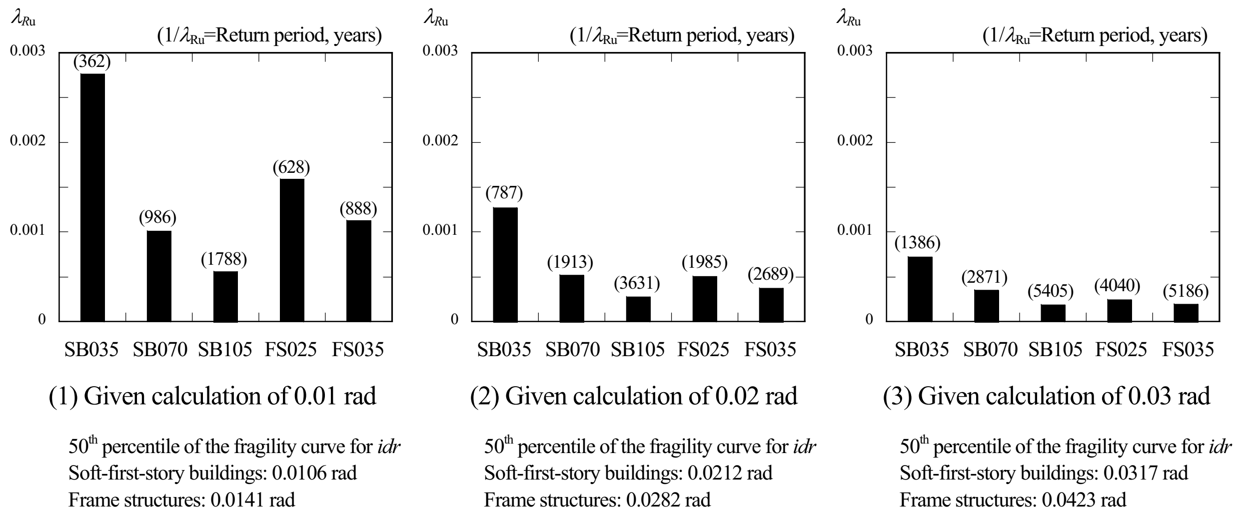


Fig. 8 Mean annual frequencies of exceeding safety limit states

$$P(\text{response} > \text{limit state}) = 1 - \exp(-\lambda_{Ru} \cdot t) \quad (9)$$

The results given 50 years are shown in Table 2. The design based on the probability can be conducted referring to this matrix. For example, if the design calls for less than 2 % probability of exceeding the limit states in 50 years, the soft-first-story building with the yield strength coefficient of 0.35 fails, and the soft-first-story building with the yield strength coefficient of 0.70 can succeed with the estimation of the deformation capacity of 0.03 rad. For the soft-first-story building with the yield strength coefficient of 1.05, the performances are equivalent to that of the frame structure with the yield strength coefficient of 0.35.

Table 2 Probabilities of exceeding safety limit states in 50 years

Cases	R_{u_cal} =0.01 rad	R_{u_cal} =0.02 rad	R_{u_cal} =0.03rad
SB 035	12.9 %	6.2 %	3.5 %
SB 070	4.9 %	2.6 %	1.7 %
SB 105	2.8 %	1.4 %	0.9 %
FS 025	7.7 %	2.5 %	1.2 %
FS 035	5.5 %	1.8 %	1.0 %

R_{u_cal} : Ultimate deformation capacity by AIJ's guideline

4. Conclusions and Future Direction

This study focused on the seismic performances of the soft-first-story buildings, which are in great demand in urban areas. The seismic performances were estimated through the probabilistic approach and compared to those of the typical frame structures.

The maximum interstory drift ratio IDR_{max} of the soft-first-story building at given the peak ground acceleration *PGA* is effectively reduced by the increase of the yield strength. However, based on the median curves of the relationships between *PGA* and IDR_{max}, the increments of IDR_{max} with the increments of *PGA* become larger in the range of over 0.04 rad, which corresponds to the yield, although the frame structures keep almost linear relationships. Thus, the shapes of curves of *PGA* and IDR_{max} are significantly influenced by the mechanism type.

The mean annual frequency of IDR_{max} exceeding the specified value *idr*, which is obtained from the results of the dynamic response analyses mentioned above and the seismic hazard curve based on *PGA*. In the seismic response hazard curves (mean annual frequency v.s. *idr*), the mean annual frequencies decrease with the increase of *idr*, and the tendency is more significant in the frame structures than in the soft-first-story buildings. That is to say, the frame structure has an advantage in the large deformation range relative to the soft-first-story structure.

By defining fragility curve based on the interstory drift angle at which the strength starts deteriorating, the probability of exceeding the safety limit state is obtained. The soft-first-story buildings can correspond to the safety of the typical frame structures with the yield strength of 0.35 and can show less than 2 % probabilities of exceeding the limit states in 50 years, on the condition that the soft-first-story building with the combination of yield strength coefficient of 0.70 and the deformation capacity estimation of 0.03 rad, or with the combination of yield strength coefficient of 1.05 and the deformation capacity estimation of 0.02 rad.

However, it should be noted that these performances are derived from the limited condition. That is say, the redundancies that come out after the safety limit state are not reflected in this estimating method. The increment of the response to that of *PGA* is more significant in the soft-first-story buildings than in the frame structures at around the safety limit state. The next step of this study is to identify the redundancies by using the models that reflect the deteriorations of the strength and to estimate real collapse capacities.

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ピロティを有する鉄筋コンクリート造建物の確率論的耐震性能評価

長江拓也・吹田啓一郎・中島正愛

要旨

本研究では，6 階建てピロティ構造の最大層間変形角に対する年間超過度数，および地震応答がピロティ柱の限界部材角に達する50 年超過確率を検証した。ピロティ構造に対して，典型的な降伏時ベースシヤ係数0.35 のフレーム構造と同等の安全性を確保するには, 終局強度型設計指針（日本建築学会, 1990）に従って, 層降伏時のベースシヤ係数 1.05 の場合に柱の限界変形角 0.02 rad 以上, 層降伏時のベースシヤ係数 0.7 の場合は，限界変形角 0.03 rad 以上を確保する必要がある。

キーワード:ピロティ建物，降伏強度，変形性能，最大層間変形角，安全限界状態，確率